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SEISMIC EVALUATION OF THE CANADIAN NATIONAL WAR MEMORIAL USING NONLINEAR GAP ELEMENTS

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ABSTRACT: The paper presents an innovative modelling technique employed on the seismic evaluation of the historic and iconic Canadian National War Memorial Monument in Ottawa, Canada. The War Memorial is a large arched cenotaph formed with stacked granite blocks. Construction of the War Memorial was completed in 1939. The monument is approximately 23 m tall and rests upon a mass concrete foundation which bears directly on bedrock. The monument supports 2 large bronze statues. The upper statue is approximately 4.4 m tall located at the top of the arch apex. Lateral resistance to overturning and sliding under seismic loading is achieved through self-weight and friction. Additional resistance to sliding is provided by bronze dowels installed at the block interfaces. As no tensile interconnection between the blocks exists, the global resistance of the cenotaph and the individual resistance of the granite blocks to overturning are achieved through self-weight alone. Under seismic loading, it is expected that the blocks will exhibit a rocking behaviour. To capture the rocking behaviour of the granite blocks, the mortar joints were modelled as nonlinear gap elements interconnecting the finite element shell objects that represent the granite blocks. Gap elements typically transfer compression forces only. The gap elements in this analysis were configured to transfer horizontal shear resistance only when subjected to axial compression forces. Due to the nonlinear response created by the rocking behaviour of the individual granite blocks, nonlinear time-history analysis was adopted as the method of analysis. Analytical models of the cenotaph were created using ETABS Nonlinear. The War Memorial was evaluated for seismic resistance based on the provisions of the 2010 National Building Code of Canada. The time histories run in this analysis indicate that there is very little lateral instability.

Keywords: Seismic Evaluation, Historic Masonry, Monument, Nonlinear Gap Element, Rocking Behaviour.

1. Introduction

Construction of the National War Memorial was completed in 1939. It consists of a large arched cenotaph formed with stacked granite blocks. The monument is approximately 23 m tall from the base of the foundation to the top; the arch is approximately 8 m tall. The cenotaph rests upon a mass concrete foundation which bears directly on bedrock.

The monument's lateral resistance to overturning and sliding under seismic loading is achieved through self-weight and shear friction. Additional resistance to sliding is provided by bronze dowels installed at the block interfaces.

The monument structure supports 2 bronze statues. One statue is located at the base of the cenotaph, and sits directly on top of the mass concrete foundation between the legs of the arch. The other is an approximately 4.4 m tall statue located at the top of the arch apex.

2. Analysis

2.1. Seismic Hazard

The provisions of the 2010 National Building Code of Canada (NBCC) requires the design of new buildings to include for the effects of earthquakes. The objectives of the 2010 NBCC with respect to earthquake resistant design are:

1) To protect the life and safety of building occupants and the general public as the building responds to strong ground shaking.
2) To limit building damage during low to moderate levels of ground shaking.
3) To ensure that post-disaster buildings can continue to be occupied and function following strong ground shaking, though minimal damage can be expected in such buildings.

In the context of this investigation into the existing National War Memorial Monument, it is proposed that only objective (1), listed above, is directly applicable. The monument's level of resistance to collapse under strong ground motion is adopted as the measure of compliance with the general public life safety objective.

According to the 2010 NBCC, strong ground motion is defined as having a probability of exceedance of 2% in 50 years at the median confidence level. This corresponds to a 1 in 2500 year earthquake.

Although stronger ground shaking than this could occur, it would be economically impractical to design for such rare ground motions. Therefore, a ground motion having a probability of exceedance of 2% in 50 years is termed as the maximum earthquake ground motion to be considered. More simply, it is termed as the design ground motion.

2.2. Design Ground Motion

The design ground motion for a structure is expressed in the 2010 NBCC as a base acceleration. The base acceleration value is a function of the specific natural period of vibration of the structure. The 5% Damped Spectral Response Acceleration values for Ottawa (City Hall) for natural periods of vibration of 0.2, 0.5, 1.0, 2.0 seconds are shown in Table 1 below. Peak Ground Acceleration (PGA) is also included.

Table 1 – Spectral Response Acceleration Values for Ottawa (City Hall)

2010 NBCC - Values for 2% Probability Exceedence in 50 Years				
PGA	**S_a(0.2)**	**S_a(0.5)**	**S_a(1.0)**	**S_a(2.0)**
0.32	0.64	0.31	0.14	0.046

Note**:** All values are in decimal percentages of g (acceleration due to gravity).

The 2010 NBCC uses site coefficients F_a and F_v to modify the above spectral values to account for the specific site soil conditions. A geotechnical investigation was conducted, and the Site Classification for seismic site response has been determined to be Site Class B. The structural analysis has been based on this Site Classification. The acceleration-based site coefficient and velocity-based site coefficient are calculated to be F_a = 0.856 and F_v = 0.640, respectively. The resulting design spectral response acceleration values for Ottawa (City Hall), Site Class B, for periods of natural vibration of 0.2. 0.5, 1.0, 2.0, and 4.0 seconds are given in Table 2 below:

Table 2 – Design Spectral Response for Ottawa (City Hall), Site Class B

2010 NBCC - Values for 2% Probability Exceedance in 50 Years				
S(0.2)	**S(0.5)**	**S(1.0)**	**S(2.0)**	**S(4.0)**
0.548	0.198	0.090	0.029	0.015

Note**:** All values are in decimal percentages of g (acceleration due to gravity).

2.3. Structural Modelling

Analytical models of the cenotaph were created using ETABS Nonlinear. Two-dimensional models were created to analyse the cenotaph in its two primary axes: parallel to the arch (referred to as E-W direction), and perpendicular to the arch (N-S direction).

In the N-S direction, a simplified analysis modelling only one half of the cenotaph arch was used. In the E-W direction, the full arch was modelled. Figure 1 below displays a 3D extrusion of the E-W direction ETABS model.

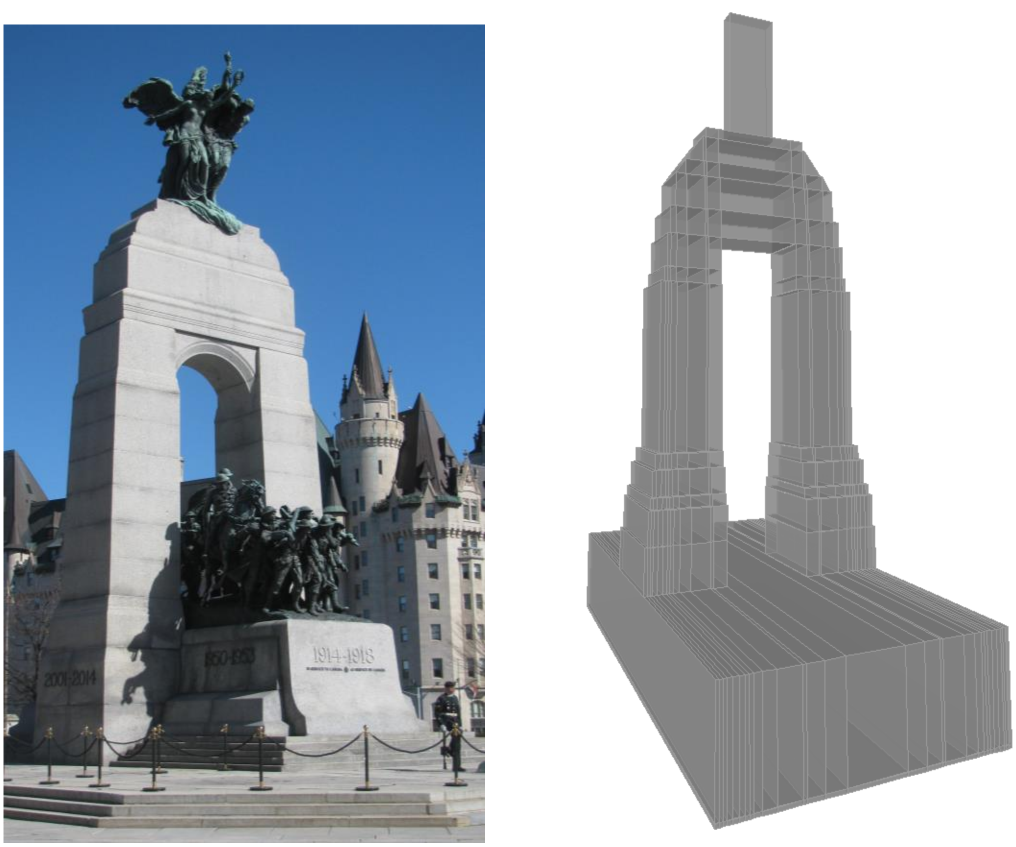

Figure 1 – Photo and ETABS Model of the Cenotaph

2.3.1. Granite Blocks

The granite blocks that form the cenotaph were modelled using finite element shell objects. The elements were assigned elastic material properties as outlined in Table 3 below:

Table 3 – Granite Material Properties

Modulus of Elasticity	50 GPa
Weight	26.5 kN/m^3
Poisson's Ratio	0.1

As no tensile interconnection between the blocks exists, the global resistance of the cenotaph and the individual resistance of the granite blocks to overturning are achieved through self-weight alone. Under seismic loading, it is expected therefore that the blocks will exhibit a rocking behaviour (i.e. the mortar joints between blocks will open at the block-to-block interfaces.)

2.3.2. Mortar Joints and Bronze Dowels

To capture the rocking behaviour of the granite blocks, the mortar joints were modelled as nonlinear gap elements interconnecting the finite element shell objects that represent the granite blocks. Gap elements typically transfer compression forces only. Since the bronze dowels prevent the blocks from sliding, the gap elements in this analysis were also configured to transfer horizontal shear resistance, but only when subjected to axial compression forces. Figure 2 below shows a segment of a time-history force plot for one of the gap elements used in the model.

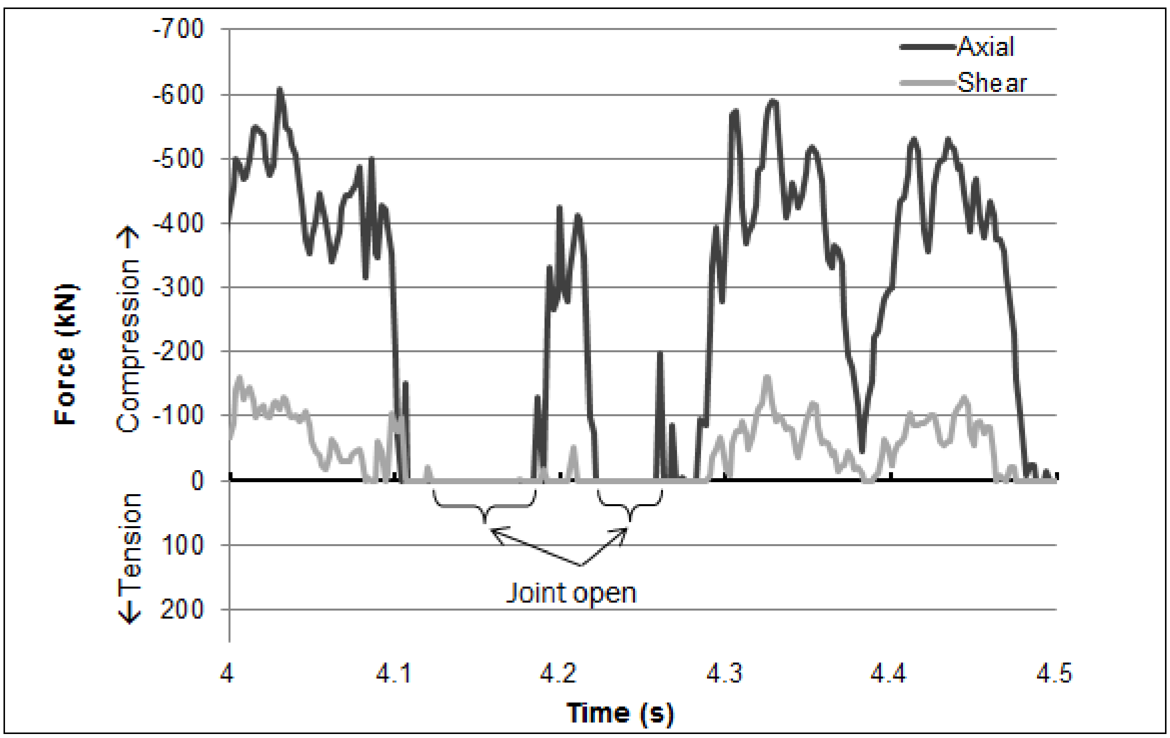

Figure 2 – Gap Element Behaviour

The time-history force plot illustrates that the gap element transfers vertical and horizontal forces only when the joint is closed (i.e. under compression).

The gaps were modelled such that when closed, they were several orders of magnitude stiffer than the adjacent shell elements. The gaps were also assigned a negligible amount of mass and 0% effective damping.

If the bronze dowels were not present, a different approach to the gap elements would be necessary to evaluate the effects of sliding and dynamic friction.

2.3.3. Bronze Statues

The monument structure supports 2 bronze statues. One statue is located at the base of the cenotaph, and sits directly on top of the mass concrete foundation between the legs of the arch. The other statue is located at the top of the arch apex.

As the lower statue is supported directly on the mass concrete foundation, it has no influence on the response of the monument to seismic loading, and was thus not included in the analytical model. The upper statue does influence the response of the monument, so a representation of its mass was incorporated in the analytical model.

2.4. Method of Analysis

Due to the nonlinear response created by the rocking behaviour of the individual granite blocks, nonlinear time-history analysis was adopted as the method of analysis. For the purpose of scaling the time-history records, equivalent static base shears were also evaluated. Discussion on the time-history records and scaling methods is presented in the following sections.

2.4.1. Time History Functions

Simulated time history records with a probability of exceedance of 2% in 50 years and their associated response spectra were obtained from seismologist Prof. Gail Atkinson on the website: *www.seismotoolbox.ca*. Guidance on the use of the data is given in the paper titled *Earthquake Time Histories Compatible with the 2005 NBCC Uniform Hazard Spectrum* (Atkinson).

The time history records were scaled such that their individual response acceleration spectrums provided a close match to the 2010 NBCC response spectrum for Ottawa (City Hall), Site Class B throughout the period range of interest (0.2 seconds to 1.0 second).

Figure 3 below shows the response acceleration spectrum of one of the raw simulated records (before scaling) and the design 2010 NBCC response spectrum for Ottawa (City Hall), Site Class B (noted as the "target" response spectrum).

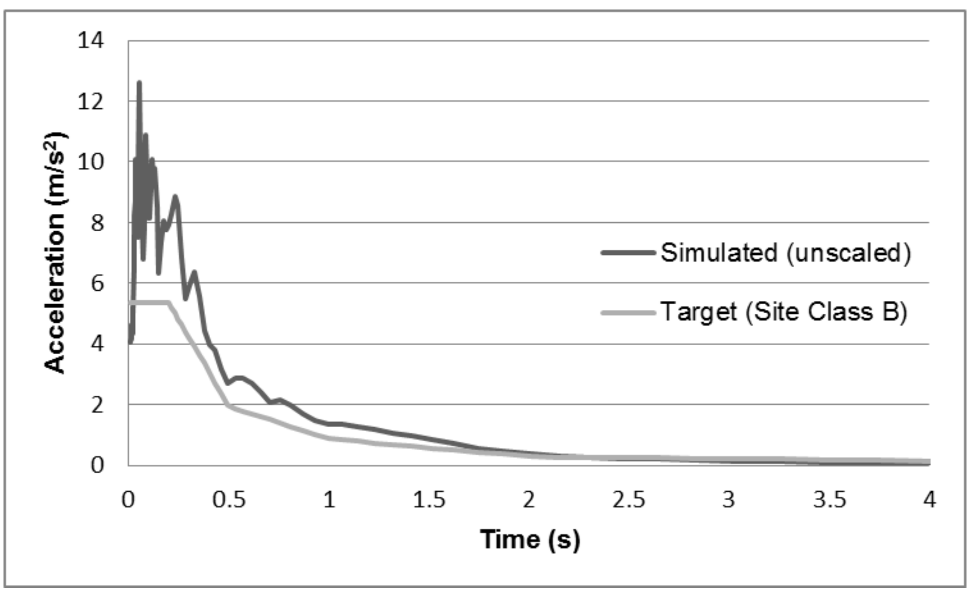

Figure 3 – Simulated Response Spectrum vs. Target Response Spectrum

Figure 4 below shows the same record, scaled to match the NBCC target spectrum for the range of 0.2 to 1.0 seconds.

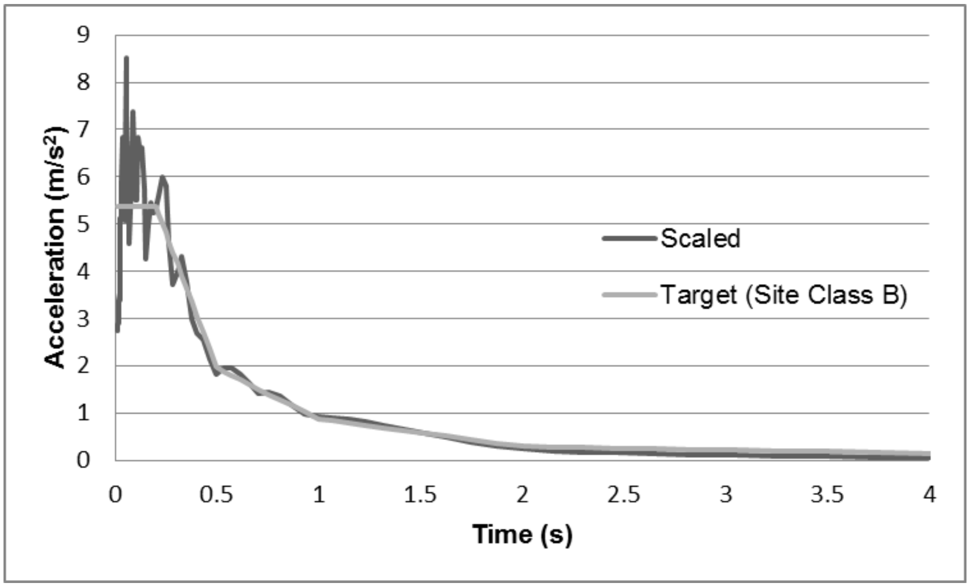

Figure 4 – Scaled Response Spectrum vs. Target Response Spectrum

For each scaled time history record, an average and standard deviation were calculated for the ratios of the target to scaled time history response spectrum acceleration values. The nonlinear time history analysis was performed using the records that had the lowest standard deviation value, i.e. the records that had the closest response spectrum match to the target response spectrum. These records were applied in both the north-south and east-west directions.

A scaled ground acceleration time history record is shown in Figure 5. The time history records were 24.6 seconds in length. Typically the most significant ground accelerations occurred over a short section of the record.

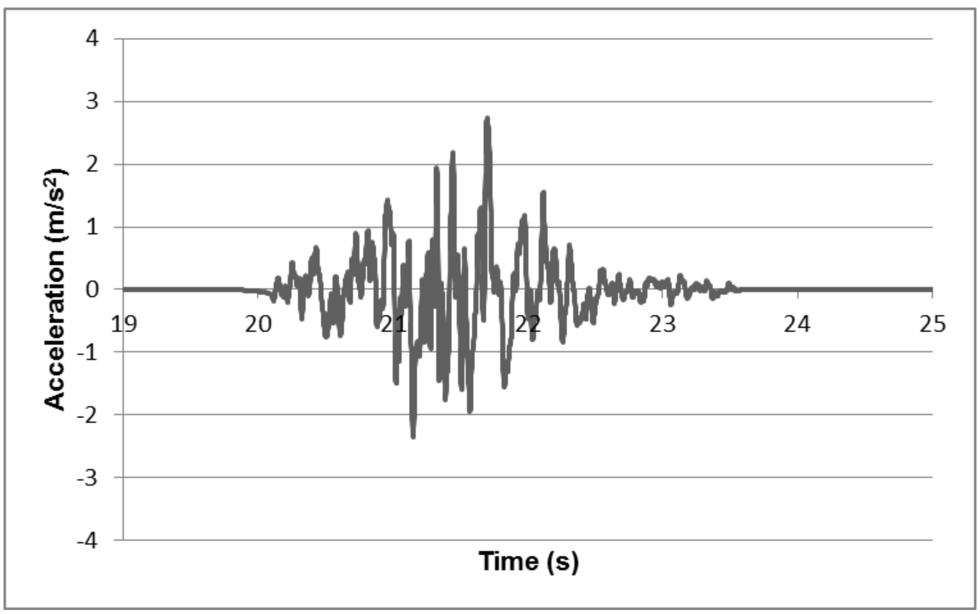

Figure 5 – Scaled Ground Acceleration Record

2.4.2. Modal Damping

It is commonly assumed that typical building structures have inherent elastic damping in the range of 2% to 5%. Due to the simplicity of the monument's structural form it is likely that it has lower elastic damping than this. For the purposes of this investigation an elastic damping value of 1% was conservatively selected. Sensitivity analysis indicated that the choice of elastic damping value did not significantly influence the overall results.

2.5. Design Base Shear - 2010 NBCC

To determine the fundamental periods of the cenotaph and the Equivalent Static Method base shears, linear versions of the models (i.e. with no nonlinear gap elements) were created.

For the purpose of this investigation, ductility and over-strength modification factors have been chosen as $R_dR_o = 1.0$. The intent is that any reductions in base shear due to rocking behaviour will occur in the nonlinear time-history analysis.

2.5.1. Fundamental Period

Modal analysis showed that the monument was a short period structure (i.e. $T<0.2s$) in both the N-S and E-W directions.

2.5.2. Equivalent Static Design Base Shear

For the purpose of comparison to the nonlinear analysis base shears, the equivalent static base shear was evaluated. The 2010 NBCC minimum Equivalent Static Method design base shear was calculated to be $V_{ESM} = 0.548W$, where W is the weight of the structure.

Table 4 – Equivalent Static Base Shears

Direction	Weight	Equivalent Static Base Shear
N-S	7,614 kN (half-model)	4,171 kN
E-W	15,576 kN	8,536 kN

2.5.3. Time History Analysis Scaling

Due to the irregular nature of the cenotaph, the 2010 NBCC dynamic analysis provisions require that the dynamic base shears be scaled such that they are greater or equal to the equivalent static base shear. Scale factors were obtained using the linear version of the model, and then applied to the nonlinear version to obtain the final results.

Table 5 – Sample N-S Direction Time History Scaling

Time History Record	Initial Linear Base Shear	Scale Factor	Scaled Linear Base Shear	Nonlinear Base Shear
6MC1NO18	1,923 kN	2.16	4,170 kN	3,117 kN
6MC2NO09	2,405 kN	1.73	4,171 kN	3,148 kN
6MC1NO42	2,110 kN	1.97	4,171 kN	4,062 kN

Table 6 – Sample E-W Direction Time History Scaling

Time History Record	Initial Base Shear	Scale Factor	Scaled Linear Base Shear	Nonlinear Base Shear
6MC1N018	3,769 kN	2.26	8,536 kN	10,058 kN
6MC2N009	8,321 kN	1.02	8,536 kN	7,319 kN
6MC1N042	3,769 kN	2.26	8,536 kN	11,760 kN

Table 5 shows that a reduction in base shears of approximately 80% occurred in a nonlinear analysis for the N-S direction. However, Table 6 shows an amplification of base shears for 2 out of 3 cases in the E-W direction.

3. Results

3.1. Structural Capacity/Demand Ratios

Structural capacity was determined to be the shear resistance due to friction between blocks and the capacity of the bronze dowels in shear at each interface. Analysis was performed to determine the friction resistance created by the normal force at the worst-case time steps. Table 7 below shows the coefficients of friction chosen for the analysis:

Table 7 – Coefficients of Friction

Material	Coefficient of Friction
Granite on Granite (Byerlee)	0.6
Foundation on Bedrock (ICC)	0.7

Table 8 below shows that in the N-S direction, the cenotaph has enough capacity to resist 100% of the 2010 NBCC seismic loads. In the E-W direction, the monument fails to resist 100% of the seismic loads; it does however have enough capacity to resist 60% of the 2010 NBCC seismic loads as required by the *RPS Policy: Seismic Resistance of PWGSC Buildings* (Public Works and Government Services Canada).

Table 8 – Block Interface Shear Capacity/Demand Ratios

Direction	Capacity/Demand Ratio
N-S	1.52
E-W	0.94

3.2. Structural Stability

Analysis showed that the 2010 NBCC seismic loads were not sufficient to cause the monument to topple. Due to the very modest storey drifts, the self-weight of the blocks is adequate to resist all global and local overturning effects. Table 9 below shows the maximum displacements and gap openings.

Table 9: Maximum Displacements and Gap Openings

Direction	Maximum Lateral Displacement (mm)	Storey Drift (%)	Maximum Gap Opening (mm)
N-S	68	0.29	15
E-W	47	0.20	12

Based on existing drawings, the installed bronze dowels at the block interfaces protrude approximately 50 mm into each block. Since the dowel length exceeds the maximum gap openings, there is no concern for loss of shear capacity.

The following Figures 6 and 7 illustrate the monument's rocking behaviour. Note that in order for them to be visible, the deflections are magnified by a scale factor of 100.

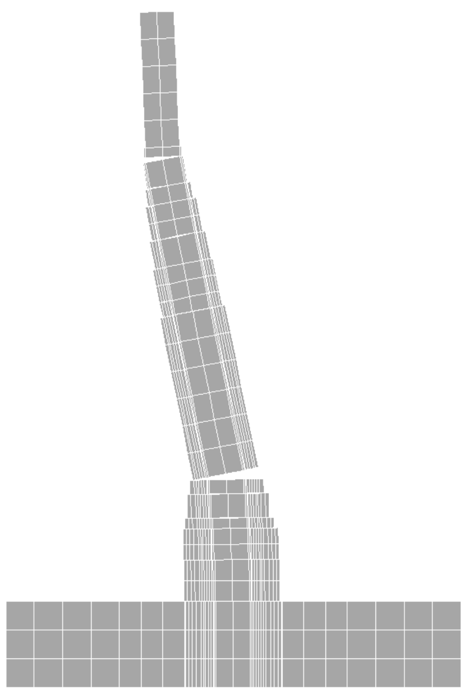

Figure 6 – N-S Direction, Deflected Shape

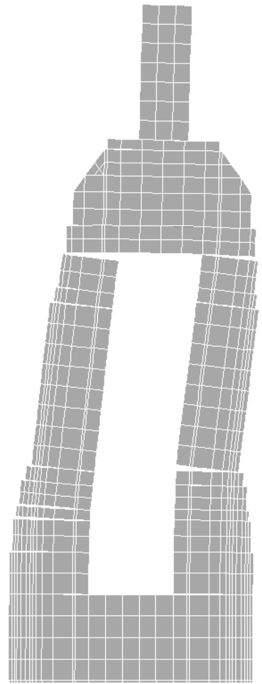

Figure 7 – E-W Direction, Deflected Shape

4. Conclusion

The method of using nonlinear gap elements to represent the rocking behaviour of granite blocks is a valid modelling technique for the National War Memorial cenotaph. The bronze dowels present between the blocks allow for the gap elements to be configured such that they transfer horizontal shear forces when subject to axial compression. As a result, the model behaved as expected by exhibiting a rocking behaviour while transferring lateral shear forces through a combination of static friction and the shear capacity of the bronze dowels.

5. Acknowledgements

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6. References

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